

## I GIRDER BEARING DESIGN FOR IMPROVED STABILITY

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### **ABSTRACT:**

*A number of bridge span-length precast prestressed I-girders approaching 200 feet have recently been completed, using 6- to 8-ft deep girders. Several factors affect I girder stability including shipping, erection and type of bearing pads. Using narrow, thick elastomeric bearing pads has resulted in reduced stability of the girders in place, before and during deck placement.*

*This paper gives an overview of the causes and remedies of precast, prestressed concrete instability during the various stages of handling, until the deck is placed and hardened. Examples of successful details are given and various resources are listed. This paper also attempts to review bearing pad design, detailing and specification. Various methods of bearing pad design are discussed and illustrated with numerical examples. The authors recommend more use of cotton duck bearing pads (CDP) in lieu of steel reinforced elastomeric pads (SREP). CDPs are stiffer and stronger. They offer excellent stability features. When combined with Teflon and stainless steel sliding interface, they provide the ability to slide without being “sheared” as is the mechanism with SREP. Oak blocking for temporary support during construction can also be used effectively to improve girder resistance to rotation. The paper also presents suggestion for considering I girder stability.*

**Keyword:** I Girder, Bearing, Cotton Duck Pads, Elastomeric Pads, bracing, stability.

## INTRODUCTION:

Several I-girder problems have been reported during construction in the past several years. In many cases the problem occurred after the girders were placed on the piers and abutments, and before the girders became composite with the deck concrete, as shown in Fig. 1. Sometimes, a girder would “tip over”, impacting the adjacent girder with the failure progressing until most girders in a span collapse and fall to the ground

A lack of bracing or other lateral restraint, combined with inappropriately designed and detailed bearing devices, and a lack of understanding of lateral bowing mechanisms, have contributed to I-girder instability before all girders in a span are tied together with the deck. This phenomenon has become more frequent in recent years due to increasing use of tall girders over long spans. Additionally, the AASHTO LRFD Bridge Design Specifications<sup>(1)</sup> have led designers in the direction of designing narrow, tall steel reinforced elastomeric bearings (SREP's), and small, relatively unstable pot bearings. It is not unheard of to see an 8 inch thick pad that is 12 inches wide under a girder with a 26-in.-wide-bottom flange, in order to meet the loading, rotation and translation requirements. Such pads are not only difficult to center under the girder, but also provide very limited lateral rotational restraint.



Fig. 1- Collapsed precast concrete girders that had been placed on circular pot bearings with Teflon and a narrow stainless steel sliding assembly.

## PRODUCTION ISSUES AND DEFINITION OF SWEEP:

Precast prestressed concrete I-girders are the solution of choice for bridge spans in the 60 to 150 feet range due to their cost effectiveness and ease of construction. Some spans as long as 350 feet have been successfully built with spliced concrete I-girders. The increasing use of 0.6-in diameter strands and high concrete strengths have allowed the use of these shapes on long spans. Recent bridges in Calgary, Alberta, Canada, utilized 2.8 m (9'-3") deep girders as long as 65 m (213 ft)

and as heavy as 138 tons. In Nebraska, Oregon and Washington State, girders as long as 185 feet have recently been constructed, and in Florida 200 feet long girders are in design.

Such escalation of spans in recent years has made girder stability issues much more important than in the past. Girder stiffness about the vertical (weak) axis is always much smaller than about the “load resisting” horizontal axis. This optimizes use of materials in the final completed condition of a bridge.

The precaster is responsible for meeting the contract specifications during production and storage in the precast plant. As such, the so called girder sweep, which is the horizontal deviation of the girder’s mid-point from its ends, should be within acceptable limits upon stripping from the forms and at the time the girder is shipped from the plant. The PCI Quality Control Manual specifies that such sweep, better identified as manufacturing sweep, shall not exceed 1/8” per 10 feet of girder length. Thus for a 160 feet long girder, the maximum allowed sweep before the girder leaves the plant is 2 in.

It should be noted that manufacturing sweep is a plant quality control measure. It is checked at the time of form stripping. It may occur due to a variety of tolerance factors such as lateral eccentricity of the prestress, either due to strand mislocation or uneven tensioning, non-uniform concrete curing, and/or a lack of symmetry of the concrete dimensions about the intended vertical axis of symmetry. Measuring sweep at the time of form stripping separates other time-dependent factors that effect sweep and allows for correction on subsequent girder castings. Sweep measurements are similar in timing and nature to initial camber measurements.

At this time, there is no mention of allowable “plumbness” for setting girders in PCI quality control documents. Until such guidance is included in future editions, it is important to attempt to set the girder such that its web at the support is vertical. This may be achieved with the aid of a carpenter’s level. A small out-of-plumbness at the supports could aggravate the lateral deformation between supports and create a potential for instability.

Some state inspectors require precasters to measure lateral deformations at the plant just before girders are shipped. This practice is encouraged, not for quality control or acceptance, but in order to have more complete records of girder behavior at various stages of its handling. Please note that these measurements should not be used for quality control or product acceptance. They are simply to provide more complete records of the condition of the girder during its entire history. Improperly stored beam could undergo thermal and creep deformations. This issue is not covered in MNL-116. It is generally covered in owner construction specifications. All parties should be clearly aware of their responsibilities as spelled out in these specifications or other contractual arrangements.

Girder lifting from the precasting bed is a critical condition requiring analysis for girder stability. Publications such as the PCI Bridge Design Manual<sup>(2)</sup> and papers by Muller<sup>(3)</sup>, Imper and Lazlo<sup>(4)</sup>, and Mast<sup>(5-7)</sup> give in detail how to analyze for girder stability. Specific recommendations are given relative to stiffness and location of lifting inserts to ensure stability. It should be noted that inserts can be as much as 15 to 20 feet from girder ends and as high as several feet above the girder top flange to ensure stability. Also, in some cases, use of strand loops as lifting inserts may not provide adequate stiffness for lifting stability. See for example Fig. 2 showing a 175 ft long, 6 ft deep girder for the Platte River East Project near Omaha, NE. Note the plate lifting insert locations and elevation above the top flange.



Fig.2- Stiff plate inserts at 14 ft from the ends of a 175-ft, 6-ft-deep girder, with lift holes high above tops of reinforcement. The girders should be stored in the precast yard on rigid dunnage in a north-south direction to avoid bowing due to uneven exposure to the sun.



Fig. 3- A 9-ft, 3-in.-deep, 61 m (200 ft) long, 130 ton, NU I-girder, being shipped with a reinforcing steel truss attached to the top flange for additional stiffness. Courtesy of Con-Force Structures, a division of Armtec Limited Partnership, Calgary, Alberta, Canada.

## SHIPPING ISSUES

Lateral girder deformations beyond the manufacturing sweep in the precast plant, can increase or decrease during shipping, depending on such factors as:

- horizontal curves and super-elevations on the route from the plant to the bridge site

- trailer-truck stiffness
- method of attachment of the girder to the truck.

Girders that are attached at their ends and allowed to rotate relative to the cab and trailer beds are susceptible to being overturned and to having growth in the lateral deformation about the weak axis.

Girder shipping is a critical condition requiring analysis for girder stability. Again, reference should be made to the PCI Bridge Design Manual, and other publications. Specific recommendations are given relative to torsional stiffness of the vehicle axles. For the 61 m (200 ft) long girders used in the bridge on 37<sup>th</sup> Street over Fish Creek in Calgary, it was necessary to attach a horizontal steel truss to the middle 30 m (100 ft) of the top flange to increase its lateral stiffness, See Fig. 3. Other measures include pretensioning or temporary post-tensioning of the top flange. An appropriately detailed shipping analysis is essential for determining whether such measures are necessary.

## **ERECTION ISSUES**

Typical long-span girders used in bridge construction are in the 75 to 100 ton weight range. Some approach as much as 150 tons, which appears to be the economical over-the-road shipping limit in most areas at this time. Heavier pieces have been shipped by water or rail. Often the erection subcontractor uses two cranes to lift the two ends separately. This is the preferred practice for such a long heavy girder. Use of a single crane could create significant compressive force in the top flange due to inclined attachment lines and exacerbate out-of-plane deformations.

Slight misalignment of lifting inserts, variations in girder cross sectional dimensions and other factors within specification tolerances, can result in the girder bowing laterally as it is lifted by cranes. Such lateral deformations, and any additional “bowing” associated with the effects of a single crane pick, are commonly misinterpreted as part of the maximum tolerable sweep. In reality, out-of-plane deformations, or bowing, present as a girder is being handled should never be called sweep. This confuses the in-plant manufacturing sweep measured for purposes of Quality Control of the manufacturing process, with external effects on the manufactured girder that are outside the control of the precaster.

When the first girder is set in position over the bearings, it should be firmly secured against any lateral translation or rotation at the supports and accurately centered on the bearings before the girder weight is released from the crane. Because it is the first girder in the span, it should be secured to the pier or abutment support. Lack of lateral restraint can result in a lack of plumbness at the girder ends which will further aggravate out-of-plane effects. Due to the relatively small moment of inertia about the girder’s weak axis, and especially if the girder is “top heavy” with a relatively large top flange, the eccentricity of the girder weight at mid-length can be large enough to cause tilting of the entire girder. Having the girders in place for a period of time, even if it is a matter of hours or days, can result in magnification of lateral deformations due to concrete creep and wind or sun effects.

Lateral deformation present in a girder after erection can be caused by non-level bearings, or by narrow bearings if the girder’s center of mass is eccentric to a line through the centerlines of the bearings. A convenience of bearings that are essentially full-width of the girder bottom flange is that a visual check for proper positioning of the bearing is generally simpler, as any misalignment on the bearing becomes more critical as the bearing gets narrower. As a general principal during

erection of bridge girders, a laterally deflected girder should tend to straighten itself if it is supported on sufficiently rigid, level bearings.

### PREVENTION OF LATERAL TRANSLATION OR ROTATION DURING AND AFTER ERECTION

Ultimately the erector/contractor must establish suitable support conditions for the girders prior to advancing with placement of deck formwork. As mentioned previously, bracing girders to their respective substructures and to one another are two methods of establishing suitable support of plumb girders. However, “suitable support” must be considered in view of girder dimensions, bearing type, bearing performance requirements and, most importantly, with an understanding that an interaction exists between the behavior of the girder and the bearing.

For example, a Nebraska NU girder has a bottom flange width of 38.3 in. which is much wider than the AASHTO I-girders and the AASHTO-PCI Bulb Tees which range from 1 ft 4 in. to 2 ft 4 in. Consequently, Nebraska and a number of other states have found wide cotton duck pads (CDP’s), which are much stiffer than steel reinforced elastomeric bearings (SREP’s) to be quite effective.

The pads are made equal to the full girder width, or girder width less 1 ½” to allow for the two ¾-in. chamfers. In addition, in Nebraska the base (shoe) plate that is embedded in the girder at the time of precasting is field welded to a 40” wide, 1 ½” thick loose sole plate; see Fig. 4. Welding is required to be completed before the crane is released. This practice has allowed placement of all girders in a span over the interstate highway system in only one night. With these details and provisions, immediate bracing has not been necessary.

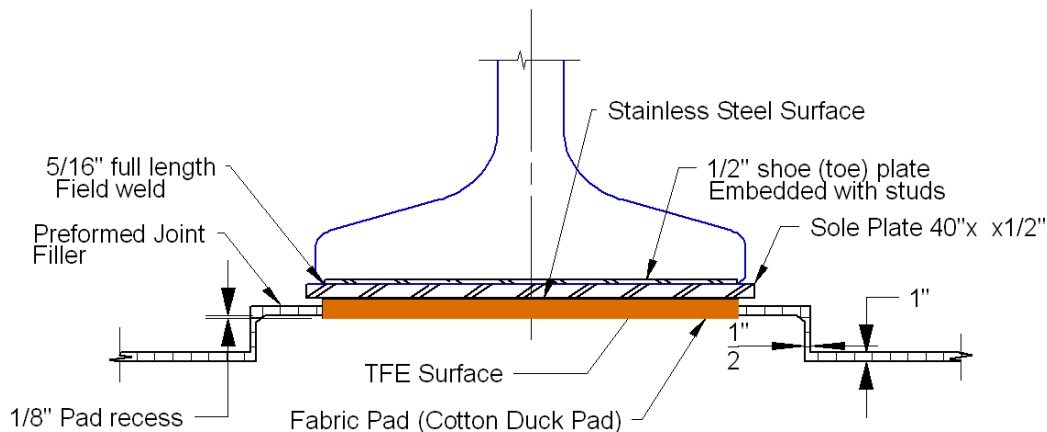


Fig. 4- Standard Nebraska bearing details for expansion bearings, using stiff wide fabric pads and separate Teflon/stainless steel sliding assembly. Stepped pedestals and concrete diaphragms transversely restrain the girders.

Note that the bottom flange width of the Nebraska (NU) I-girder is 38.3” which is much wider than the AASHTO I-girders and the AASHTO PCI Bulb Tees which range from 1 ft. 4 in. to 2 ft. 4 in. AASHTO I-girders and BT’s should still have a full-width bearing pad, and a wide thick sole plate. Where significant movement is expected in expansion bearings, it is preferable to place Teflon and stainless steel sliding interface, rather than to count on the shearing SREPs. For relatively narrow bottom flanges, as is the case in some AASHTO girders, it is often necessary to use sole plates with slotted holes and anchor bolts as shown in Fig. 5. Similarly, pot bearings are narrow and their use would contribute to girder lateral instability.

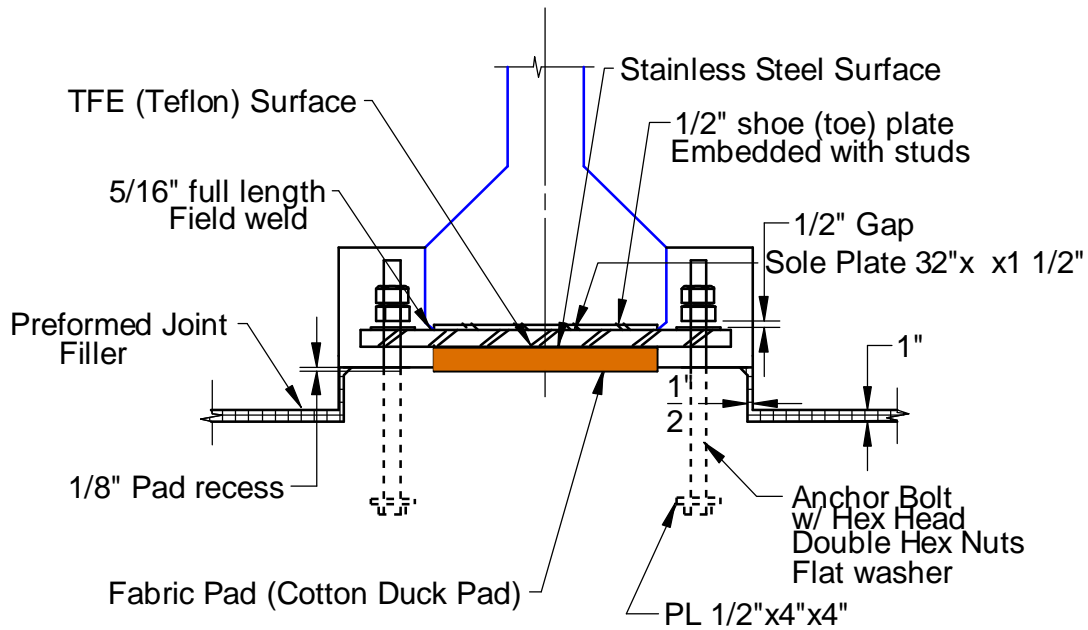


Fig. 5- Use of anchor bolts at an expansion bearing for a relatively narrow bottom flange for lateral stability

Another approach is to exercise a thought probably attributable to a wily, bridge maintenance inspector - the best bearing device is no bearing. Several Northwest states come close to this practice. At fixed (against translation) bearings, the girders are set on oak, or some other hardwood blocking, see Fig. 6. If the girder/pier joint is designed as a pin, the wood is saw-cut or otherwise removed after the diaphragm concrete has hardened. For, a monolithic “integral” joint, the hardwood blocking may be left in place as the diaphragm concrete flows around it and forms a continuous medium with the rest of the diaphragm. The same practice of using oak wedges for stability could be used to support girders with bearings that are narrower than the bottom flange, if for example live loading requires that the bearing pad be narrower than the bottom flange width.



Fig. 6- Precast girder set on oak blocking before the entire girder end is enclosed in a concrete cross beam.

The photo in Fig. 7 is an example of poor design and detailing of the bearing area with respect to prevention of translation and rotation of the girder end during and after erection. This design may have been driven in part by the directions given in the current AASHTO LRFD Specifications as discussed in the next section. Confronted with such a detail an erector/contractor may do well to integrate a Northwestern approach by placing hardwood blocking and wedges, or some other crush resistant material, adjacent to the narrow bearing until the concrete deck is cured.



Fig. 7- Example of a narrow, tall bearing pad that must be given attention by all project stakeholders

This report does not state what level of lateral deformation is tolerated without further action and which requires girder restoration to plumbness. In most situations, quick action is required once it is determined that the girder plumbness must be restored. Fig. 8 shows a bridge with deformed girders that were restored to plumbness.



Figure 8- Lateral deformation subsequently removed by plumbing beam ends and reattaching cross bracing to the straightened girder.

### **GUIDELINES FOR IMPROVED DESIGN AND DETAILING OF BEARINGS:**

The AASHTO LRFD Bridge Design Specifications, Fourth Edition, 2007, contains the requirements for design of elastomeric bearings in Articles 14.7.5 (Method B) and 14.7.6 (Method A). Method A is more approximate than Method B which requires special testing before the pads are approved for installation. The design using Method B, or even the approximate Method A, is quite complex and contains conflicting demands.

Both methods assume that the bearing pads are required to accommodate horizontal and rotational movements at the bearing, while limiting vertical movement, to avoid a frequent “bump” at the deck level and the resulting driver discomfort and deck expansion joint deterioration. With respect to translational movements the Specifications require a pad thickness that is at least twice the expected translation to guard against “rollover” of the bearing edges and “delamination” of the layers of elastomer from the internal shim plates in SREP’s.

A minimum compressive stress is required for the purpose of prohibiting tension in the pads due to girder rotation. Inclusion of a minimum rotation of 0.005 radians for a random combination of construction tolerances, coupled with a maximum compressive stress limit produce such a tightly bounded solution envelop that narrow, tall bearings are the most likely candidates to fall within

the design envelope. Although “stability” requirements are present, the current Specifications are silent with respect to lateral deformations that may occur and their contribution to girder instability during construction. To account for stability concerns, some states supplement the AASHTO LRFD Specifications with a requirement that the thickness not exceed a certain limit (e.g. 5 inches in Nebraska).

For favorable “system” performance of the girder and SREP, the SREP should be as wide as possible, as short as possible (along the girder axis) and as thin as possible. These proportions provide improved lateral stability and improved vertical stiffness. Given the multitude of constraints present in the current LRFD Specifications, to achieve more favorable system performance, some engineers prefer that expansion requirements be accommodated by other means than the shearing deformation abilities of the SREP.

Typically this may involve a detail such as shown in Fig. 4 with a relatively stiff CDP coupled with a Teflon sliding plate. However, as presented in Table 14.6.2-1 – Bearing Suitability, use of CDP’s appears to be discouraged. The table is a furtherance of the past practice of viewing details component by component rather than acknowledging that “system” performance must ultimately be the measure of suitability. Recommendations for design of CDP according to the practices of the Nebraska Department of Roads and the results of a recent research program by Lehman et al<sup>(9)</sup> for the Washington Department of Transportation are given in a separate section.

A recent study by Stanton et al. (NCHRP 12-69, Report 596)<sup>(8)</sup> addresses some of these issues and offers considerable improvements. They recommend allowing tension between the pads and the supporting materials and also allow for larger allowable compressive stress resistance. The recommendations of NCHRP 12-69 have already been approved at the AASHTO meeting in May 2008 in Omaha, NE, for inclusion in the Interim 2009 to the LRFD Specifications. This revision will provide much needed relief.

According to NDOR policies, the following guidelines should be used in selecting the type of bearing pad to use in design:

- (1) Cost and size are the two factors generally used in selecting a bridge bearing. Because of the rapid design using spreadsheets, designers are advised to develop design using both SREP and CDP and then select the better one.
- (2) For fixed bearings with moderate loads there is little cost difference between CDP, plain elastomeric pads, and moderately sized SREP with a minimal number of steel shims.
- (3) For expansion bearings with movement up to 2.5 in., there is little difference between CDP with PTFE/stainless steel and SREP.
- (4) For expansion bearings with movement over 2.5 in., CDP with PTFE/stainless steel is generally the appropriate solution, due to the requirement that pad thickness cannot exceed 5 in.
- (5) All CDP and SREP should be recessed 1/8” into the girder seat.
- (6) To accommodate horizontal movement a 3/32” (0.09375 in.) thick, low friction, virgin, unfilled Polytetrafluorethylene (PTFE) sheet shall be bonded to the upper surface of the CDP. PTFE is not allowed to be used with SREP.
- (7) The stainless steel plate should be 0.07 to 0.078 in. thick. It should have a length along the girder axis of the larger of 11 in. and (pad length + movement due to temperature, creep and shrinkage). Its width should equal the pad width plus ¾ in.

#### **COTTON DUCK PAD DESIGN:**

**I. According to NDOR policies:**

- (1) Pads with the least volume are generally the least expensive. Minimizing the pad length would result in the least volume as the thickness is a function of the length. It also provides a relatively wide pad which aids in transverse stability.
- (2) All CDP bearings are designed assuming a total rotation of 0.015 radians. This results in a pad thickness of at least (pad length/5.3)
- (3) Maximum pad thickness = 3 in.
- (4) Minimum pad length = 4 in.
- (5) Allowable pad compressive stress due to dead plus live loads = 1.5 ksi
- (6) Design steps: (a) pad area = load/1.5; (b) length = area/width, with width assumed as wide as geometry allows; (c) thickness = length/5.3.

**II. According to University of Washington Research**

- (1) Maximum average compressive stress due to total load = 3 ksi and due to LL = 2 ksi.
- (2) Minimum thickness must satisfy each of the following three criteria
  - (a) To control compressive strain:  $t_p \geq \frac{100_s L}{[4 - \sigma_s]}$
  - (b) To limit total rotation:  $t_p \geq \frac{12.50_s L}{\sigma_s}$
  - (c) To limit live load rotation:  $t_p \geq \frac{500_L L}{\sigma_s}$

**BEARING DESIGN EXAMPLES:**

The following examples demonstrate the options available in design of bearing pads in accordance with various versions of the design methods in the *LRFD Bridge Design Specifications*.

**Example 1, Comparison of Methods of steel reinforced elastomeric pad (SREP) design**

A simple, 160 ft span, bridge, located in Lincoln, Nebraska is considered. NU (Nebraska) 2000 (78 in.) prestressed concrete I girder with 10 ft center-to-center spacing and an 8-in. deck. The total horizontal movement including thermal, shrinkage and creep is 1.5 in. The NU girders bottom flange is 38.3 in. wide.

Dead load reaction,  $R_D = 150$  kips. Live load reaction,  $R_L = 100$  kips

For maximum girder stability, it is desired to have a bearing width = girder width minus 2-3/4 in. chamfers = 36.8 in. The rotation of the member end = 0.01 radian. The following methods were used in design: Approximate Method A and detailed Method B of the current AASHTO, and Methods A and B of the soon to be published 2009 Interim to AASHTO.

The detailed calculations using method B are shown to illustrate the steps that must be taken in design and the complexity of design.

**Using Method B of the current AASHTO.**

Select a rectangular, steel-reinforced elastomeric pad (SREP) with a Shore A Durometer hardness of 50 points. The shear modulus  $G$  is assumed to have a minimum value of 95 psi and a maximum value of 130 psi. Since this is an expansion bearing, the following stress limits apply:

$$\sigma_s \leq 1.66GS \leq 1.6 \text{ ksi} \quad [\text{LRFD Eq. 14.7.5.3.2-1}]$$

$$\sigma_L \leq 0.66GS \quad [\text{LRFD Eq. 14.7.5.3.2-2}]$$

For preliminary sizing, the stress in the bearing can be computed using the equation:

$$\sigma_s = \frac{P_D + R_L}{LW}$$

Where  $R_D$  = dead load reaction,  $R_L$  = live load reaction,  $L$  = length of bearing pad, and  $W$  = width of bearing pad. Try  $L = 6.0$  in. and compute the stress:

$$\sigma_s = \frac{P_D + R_L}{LW} = \frac{150 + 100}{6 \times 36.8} = 1.13 < 1.6 \text{ O.K.}$$

Determine the required shape factor,  $S$ , using LRFD Eq. 14.7.5.3.2-1, with  $G = 95$  psi:

$$S \geq \frac{\sigma_s}{1.6G} = \frac{1.13}{1.6 \times 0.095} = 7.43$$

Note that the low value in the range of  $G$  values is used here to obtain the maximum shape factor.

Determine maximum layer thickness,  $h_{\max}$ , by solving LRFD Eq. 14.7.5.1-1:

$$h_{\max} \leq \frac{LW}{2S(L+W)} = \frac{6 \times 36.8}{2 \times 7.43 \times (6 + 36.8)} = 0.395$$

Select 0.25 in. internal layer thickness. Thus,  $h_{ri} = 0.25$  in.  $\leq h_{\max} = 0.395$  in.

Compute final shape factor:

$$S = \frac{LW}{2h_{\max}(L+W)} = \frac{6 \times 36.8}{2 \times 0.25 \times (6 + 36.8)} = 10.317 > 7.43 \text{ O.K.} \quad [\text{LRFD Eq. 14.7.5.1-1}]$$

Check the live load stress according to [LRFD Eq. 14.7.5.3.2-2]:

$$\sigma_L \leq 0.66GS = 0.66 \times 0.095 \times 10.317 = 0.646 \text{ ksi}$$

$$\text{Actual } \sigma_L = \frac{R_L}{LW} = \frac{100}{6 \times 36.8} = 0.45 \text{ ksi} < 0.646 \text{ O.K.}$$

Total thickness of elastomer must be at least twice the total movement:

$$h_{rt} \geq 2\Delta s = (2)(1.5) = 3.0 \text{ in.} \quad [\text{LRFD Eq. 14.7.5.3.4-1}]$$

Use eleven internal layers at 0.25 in. plus two cover layers at 0.125 in.. Thus, the total elastomer thickness:  $h_{rt} = 2.75 + 0.25 = 3.0$  in. OK

[LRFD Eq. 14.7.5.3.5-1] equation for combined compression and rotation must be satisfied to ensure that no point on the bearing will undergo a net upward movement (uplift). Note that this limit has been liberalized in the 2009 provisions. The total rotation is 0.01 radians.

$$\sigma_s > 1.0GS \left( \frac{\theta_s}{n} \right) \left( \frac{B}{h_{ri}} \right)^2 \quad \text{[LRFD Eq. 14.7.5.3.5-1]}$$

Where  $B = L = 6.0$  in.  $n = 12$  layers of elastomer; (11 internal layers and 2 cover layers, each counting as one-half of an internal layer).  $G$  in this check should be the high value of the range, 130 psi.

$$\sigma_s > 1.0 \times 0.130 \times 10.317 \left( \frac{0.01}{12.0} \right) \left( \frac{6}{0.25} \right)^2 = 0.643 \text{ ksi}$$

$$\sigma_s = 1.13 > 0.643 \text{ O.K}$$

Thus, uplift will not occur.

Check that the maximum compressive strain at the edge is not excessive. Note that  $G = 95$  psi for this check:

$$\sigma_s < 1.875GS \left[ 1 - 0.2 \left( \frac{\theta_s}{n} \right) \left( \frac{B}{h_{ri}} \right)^2 \right]$$

$$\sigma_s < 1.875 \times 0.095 \times 10.317 \left[ 1 - 0.2 \times \left( \frac{0.01}{12.0} \right) \left( \frac{6}{0.25} \right)^2 \right] = 1.66 \text{ ksi}$$

$$\sigma_s = 1.13 < 1.66 \text{ ksi OK}$$

Stability check: If bearings satisfy the following equation, no further investigation of stability is required.

$$2A \leq B \quad \text{[LRFD Eq. 14.7.5.3.6-1]}$$

Where

$$A = \frac{1.92 \left( \frac{h_{ri}}{L} \right)}{\sqrt{1 + \frac{2.0L}{W}}} = \frac{1.92 \left( \frac{3.0}{6.0} \right)}{\sqrt{1 + \frac{2.0 \times 6}{36.8}}} = 0.83 \quad \text{[LRFD Eq. 14.7.5.3.6-2]}$$

$$B = \frac{2.67}{(S + 2.0) \left( 1 + \frac{L}{4W} \right)} = \frac{2.67}{(10.317 + 2.0) \left( 1 + \frac{6.01}{4 \times 36.8} \right)} = 0.208 \quad \text{[LRFD Eq. 14.7.5.3.6-3]}$$

$$2A = 2 * 0.83 = 1.66 > B \quad \text{NG}$$

Alternatively, assume that the bearing detail is fixed at the other girder end, use [LRFD Eq. 14.7.5.3.6-5] to check  $\sigma_s$ , using  $G = 95$  psi:

$$\sigma_s \leq \frac{GS}{A - B} = \frac{0.095 \times 10.317}{0.83 - 0.208} = 1.57$$

$$\sigma_s = 1.13 < 1.57 \text{ O.K.}$$

Reinforcement design: Use mild steel with 36 ksi yield stress ( $F_y$ ) and a 24 ksi fatigue limit ( $\Delta F_{TH}$ , from LRFD Table 6.6.1.2.5-3, Category A). Check the strength of the plates at the service limit state:

$$h_s \geq \frac{3.0 h_{\max} \sigma_s}{F_y} = \frac{3.0 \times 0.25 \times 1.13}{36} = 0.0235 \text{ in.} \quad [\text{LRFD Eq. 14.7.5.3.7-1}]$$

Check fatigue requirements for the steel plates:

$$h_s \geq \frac{2.0 h_{\max} \sigma_L}{\Delta F_{TH}} = \frac{2.0 \times 0.25 \times 0.45}{24} = 0.0094 \text{ in.} \quad [\text{LRFD Eq. 14.7.5.3.7-2}]$$

The strength equation governs. The plate thickness must be at least 0.0248 in. Select 12-gauge steel plate,  $h_s = 0.1046$  in. Thus total pad thickness =  $3.0 + 12 * 0.1046 = 4.26$  in.

**Using Method B of the soon to be published 2009 Interim to AASHTO.**

### Shear deformation

Total thickness of elastomer must be at least twice the total movement:

$$h_{rt} \geq 2\Delta_s \quad [\text{LRFD Eq. 14.7.5.3.2-1}]$$

Where

$h_{rt}$  = total thickness (in).

$\Delta_s$  = maximum shear deformation (in).

$$h_{rt} \geq 2\Delta_s = (2)(1.5) = 3.0 \text{ in.}$$

Use eleven internal layers at 0.25 in. plus two cover layers at 0.125 in.. Thus, the total elastomer thickness:  $h_{rt} = 2.75 + 0.25 = 3.0$  in. OK

Use 5.5 L \* 36.8 W pad

Combined compression, rotation and shear

$$(\gamma_{a,st} + \gamma_{r,st} + \gamma_{s,st}) + 1.75 (\gamma_{a,cy} + \gamma_{r,cy} + \gamma_{s,cy}) \leq 5 \text{ [LRFD Eq. 14.7.5.3.3-1]}$$

$$\gamma_{a,st} \leq 3 \text{ [LRFD Eq. 14.7.5.3.3-2]}$$

Where:

Subscripts “st” and “cy” indicate static and cyclic loading

$\gamma_a$  = shear strain caused by axial load.

$\gamma_r$  = shear strain caused by rotation.

$\gamma_s$  = shear strain caused by shear displacement

$$\gamma_a = (D_a * \sigma_s) / (G * S_i) \text{ [LRFD Eq. 14.7.5.3.3-3]}$$

Where:

$D_a$  = dimensionless coefficient used to determine shear strain due to axial load.

$$D_a = 1.4 \text{ (for rectangular bearing)}$$

$$\sigma_s = (R_D + R_L) / A = (100 + 150) / (5.6 * 36.8) = 1.235 \text{ ksi}$$

$$\gamma_a = (D_a * \sigma_s) / (G * S_i)$$

$$S_i = \frac{LW}{2h_{\max}(L+W)} = \frac{36.8 * 5.6}{2 * .25 * (36.8 + 5.5)} = 9.9$$

$$\gamma_a = (1.4 * 1.235) / (0.095 * 9.9) = 1.84 < 3 \text{ OK}$$

G in this check should be the low value of the range, 95 psi

$$\gamma_r = D_r * (\theta_s / n) * (L / h_i)^2 \text{ for rectangular bearing [LRFD Eq. 14.7.5.3.3-6]}$$

$D_r$  = dimensionless coefficient used to determine shear strain due to rotation.

$\theta_s$  = maximum static or cyclic service limit state rotation angle of the elastomer specified in article 14.4.2.1 (rad)

n = number of interior layers of elastomer

$$\text{In which } D_r = 0.5 \text{ [LRFD Eq. 14.7.5.3.3-7]}$$

$$\gamma_r = 0.5 * (0.01 / 11) * (5.5 / 0.25)^2 = 0.22$$

$$\gamma_s = \Delta_s / h_{rt} \text{ [LRFD Eq. 14.7.5.3.3-10]}$$

$$\gamma_s = 1.5 / 3.0 = 0.5$$

$$(\gamma_{a,st} + \gamma_{r,st} + \gamma_{s,st}) = 1.84 + 0.22 + 0.5 = 2.56 < 5 \text{ OK}$$

Stability :

Stability check: If bearings satisfy the following equation, no further investigation of stability is required.

$$2A \leq B \quad [\text{LRFD Eq. 14.7.5.3.4-1}]$$

Where

$$A = \frac{1.92 \left( \frac{h_{ri}}{L} \right)}{\sqrt{1 + \frac{2.0L}{W}}} = \frac{1.92 \left( \frac{3.0}{5.5} \right)}{\sqrt{1 + \frac{2.0 \times 5.5}{36.8}}} = 0.92 \quad [\text{LRFD Eq. 14.7.5.3.4-2}]$$

$$B = \frac{2.67}{(S_i + 2.0) \left( 1 + \frac{L}{4W} \right)} = \frac{2.67}{(9.9 + 2.0) \left( 1 + \frac{5.5}{4 \times 36.8} \right)} = 0.22 \quad [\text{LRFD Eq. 14.7.5.3.4-3}]$$

$$2A = 2 \times 0.92 = 1.84 > B$$

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Alternatively, assume that the bearing detail is fixed at the other girder end, use [LRFD Eq. 14.7.5.3.4-5] to check  $\sigma_s$ , using  $G = 95$  psi:

$$\sigma_s \leq \frac{GS_i}{A - B} = \frac{0.095 \times 9.9}{0.92 - 0.22} = 1.34$$

$$\sigma_s = 1.235 < 1.34 \text{ OK}$$

Reinforcement design:

Use mild steel with 36 ksi yield stress ( $F_y$ ) and a 24 ksi fatigue limit ( $\Delta F_{TH}$ , from LRFD Table 6.6.1.2.5-3, Category A). Check the strength of the plates at the service limit state:

$$h_s \geq \frac{3.0 h_{\max} \sigma_s}{F_y} = \frac{3.0 \times 0.25 \times 1.235}{36} = 0.0257 \text{ in.} \quad [\text{LRFD Eq. 14.7.5.3.5-1}]$$

Check fatigue requirements for the steel plates:

$$h_s \geq \frac{2.0 h_{\max} \sigma_L}{\Delta F_{TH}} = \frac{2.0 \times 0.25 \times 0.49}{24} = 0.01 \text{ in.} \quad [\text{LRFD Eq. 14.7.5.3.7-2}]$$

The strength equation governs. The plate thickness must be at least 0.0257 in.

Select 12-gauge steel plate,  $h_s = 0.1046$  in. Thus total pad thickness =  $3.0 + 11 \times 0.1046 = 4.26$  in.

A summary of the results of all four methods is given below. The pad width and thickness are kept constant for comparison purposes. Table 1 shows the various pad lengths resulting from use of different methods.

Table 1- Bearing pad length according current and the 2009 AASHTO LRFD methods

	Method A	Method B
Current (2007) AASHTO	12.0	6.0
2009 AASHTO	11.0	5.5

The results in Table 1 show Detailed Method B requires a smaller length than Method A, as expected. However, the difference is significant, indicating that Method B should be used in design to try to minimize pad length and therefore improve details and stability of the girder end.

**Example 2, Variation in SREP sizes using Current Method B:**

Table 2 demonstrates that there is no unique solution to the problem, even when a single design method is used. Here, Method B of the current (2007) AASHTO LRFD Specifications is applied. Options 1 and 2 give the same length and thickness. However, the pad in option 2 is wider and is therefore preferred for improved transverse stability, although it would be more expensive. Option 4 should be the least desirable as it involves the narrowest and thickest pad, therefore the least desirable stability features.

**Table 2- Options for the Example 1 data using current Method B**

Option	Width (in.)	Length (in.)	Thickness (in.)
1	27.00	6.00	4.26
2	36.80	6.00	4.26
3	18.00	9.00	5.32
4	15.00	10.50	6.74

Please note that several assumptions were made within this design. They include for example the elastomer hardness and layer thickness. Also, the amount of rotation designed for may be called to question depending on the girder to support details. Likely for prestressed girders under current practice, two bearings are used at each pier and live load rotation is highly restrained. Thus, the number of plausible solutions can be considerably greater than shown here. It is important, thus, to ensure good engineering

judgment relative to stability. Also, simplification of current SREP design methods would be highly desirable.

The same variability exist with 2009 AASHTO procedure and the recommend held to the selection procedure needs to be simplified.

### **Example 3, Design of cotton duck pad (CDP) design:**

A cotton duck pad (CDP) will be designed according to Nebraska Department of Roads procedure, which is extremely simple and has proven over the past twenty years to produce acceptable field performance.

The total load of 250 kips requires a pad area of  $250/1.5 = 167 \text{ in}^2$ . Given a width of 37.8 in. the required pad length =  $167/36.8 = 4.75 \text{ in}$ . Minimum pad thickness =  $\text{length}/5.3 = 1 \text{ in}$ . Thus a 37.8 by 4.5 by 1 in. CDP. Teflon and stainless steel thickness of 0.09375 in. is added to this thickness to determine the total thickness. The stiffness and shallow thickness of the CDP is expected to provide superior girder stability that the SREP solution.

The following values result when same pad is designed according to University of Washington recommendations; required area =  $250/3 = 83.3 \text{ in}^2$ , or  $100/2 = 50 \text{ in}^2$ . Use the larger area. Assume width = 36.8 in., length =  $83.3/36.8 = 2.26 \text{ in}$ . Minimum length of 4 in. will be used. Thickness of controlled by one of three factors: compression strain yields 0.17 in. Total rotation yields 0.29 in. and live load rotation limit yields 0.59 in. Thus, use a 36.8 by 4 by  $\frac{3}{4}$  in. pad.

It appears from this example that the NDOR criteria are quite conservative, yet produce reasonably sized pads even for the relatively high load considered in the example. It is the recommended method by the authors until more confidence is attained as usage of this type is increased.

## **CONCLUSIONS AND RECOMMENDATIONS:**

### **Conclusions:**

Bearing pads are important elements in supporting girders over the substructure. Pads should be as thin and as short as conditions will allow. On the other hand, pads should be nearly as wide as the bottom flange to limit the potential for lateral rotation. Method B of the 2009 Interim of AASHTO Bridge Design Specifications gives the most realistic pad size and its use should be encouraged for design of Steel reinforced Elastomeric Pads (SREP). However, the authors believe that Cotton Duck Pads (CDP) are superior for girder stability. Their design is extremely simple. Lack of coverage of this type of pad in AASHTO tends to implicitly discourage their use. For expansion type bearings, this pad type requires a separate mechanism of Teflon and stainless steel interface. This should be considered an advantage over the shearing deformation of SREPs. Experience with their use in Nebraska, Washington over the past two decades has been positive.

**Recommendations:**

The following recommendation should be considered for I girder stability in general.

- a. Owner Agency/Consultant
  - i. Establish that a viable erection bracing plan, or some alternate method to secure the girder ends in a fixed plumb condition before crane release exists before letting the project whether the means and methods are explicitly incorporated in the contract plans or not.
  - ii. Consider the costs/benefits of specify bearings that are as wide as the girder bottom flange minus the chamfers.
  - iii. Always evaluate the “system” performance of girders and their bearings and actively engage project stakeholders in coordination of a “system” performance approach on each project.
  - iv. Consider using cotton duck pads where appropriate to reduce pad deformability during erection
  - v. Consider allowing hard wood shimming pads where appropriate to reduce pad deformability during erection
- b. Precaster
  - i. Measure and record girder manufacturing sweep after removal from form; confirm compliance with allowable tolerance.
  - ii. Store girders on non-compressible dunnage on firm level ground; preferably in a N-S orientation.
  - iii. Monitor plumbness of girder webs and girder bowing in storage.
  - iv. At shipping, measure bowing of any girder that appears “bowed” and record the measurement.
- c. Shipper
  - i. Bunk girders on wide, stiff supports; follow guidelines in the PCI Bridge Design Manual
  - ii. Avoid permitting truck/trailer to rest for any length of time on terrain with cross slope, in particular at the job site prior to off-loading and during off-loading of the girder.
- d. Contractor
  - i. Handle girder only with installation lift loops noted on erection drawings as being for contractor use.
  - ii. Set girder centered on bearings
  - iii. Set girder in a vertical plumb position, using carpenter level or other means, and secure against lateral rotation, using bracing or other means, before the crane is released.
  - iv. If erected girder lateral deformation is considered excessive, seek immediate assistance of a licensed engineer in advisability of remediation. Such remediation may include additional cross frames and/or girder straightening. Be aware that bent, distorted cross frames and laterally deformed girders are indicators that unintended forces may be present in the cross frames and should therefore not be removed without first seeking advice and counsel of a licensed engineer. Disconnecting a cross frame without first considering what remains present to restrain the girder is a potential hazard.

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